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CENTRIFUGE MODELLING FOR CIVIL ENGINEERS

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Editor

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Preface

Centrifuge modelling involves testing of small scale models in the enhanced gravity field of a geotechnical Method centrifuge. This technique is particularly useful in testing materials such as soils which exhibit non-linear stress-strain behaviour and can suffer significant plastic strains. Centrifuge modelling is proven to be particularly effective in determining the failure mechanisms for a wide variety of geotechnical problems. The scale model is typically constructed in the laboratory and then loaded onto the end of the centrifuge, which is typically between 0.2 and 10 metres (0.7 and 32.8 ft) in radius. The purpose of spinning the models on the centrifuge is to increase the g-forces on the model so that stresses in the model are equal to stresses in the prototype. A geotechnical centrifuge is used to test models of geotechnical problems such as the strength, stiffness and capacity of foundations for bridges and buildings, settlement of embankments, stability of slopes, earth retaining structures, tunnel stability and seawalls. Other applications include explosive cratering, contaminant migration in ground water, frost heave and sea ice.

This book summaries a generalized design process engaged for civil engineering projects. This book is a reference tool for graduate students, researchers, and practicing civil engineers involved with geotechnical issues.

Editor

Contents

Chapter 1

Centrifuge Modeling of Rocking-Isolated Inelastic Rc Bridge Piers

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ABSTRACT

Experimental proof is provided of an unconventional seismic design concept, which is based on deliberately under designing shallow foundations to promote intense rocking oscillations and thereby to dramatically improve the seismic resilience of structures. Termed rocking isolation, this new seismic design philosophy is investigated through a series of dynamic centrifuge experiments on properly scaled models of a modern reinforced concrete (RC) bridge pier. The experimental method reproduces the nonlinear and inelastic response of both the soil-footing interface and the structure. To this end, a novel scale model RC (1:50 scale) that simulates reasonably well the elastic response and the failure of prototype RC elements is utilized, along with realistic representation of the soil behavior in a geotechnical centrifuge. A variety of seismic ground motions are considered as excitations. They result in consistent demonstrably beneficial performance of the rocking-isolated pier in comparison with the one designed conventionally. Seismic demand is reduced in terms of both inertial load and deck drift. Furthermore, foundation uplifting has a self-centering potential, whereas soil yielding is shown to provide a particularly effective energy dissipation mechanism, exhibiting significant resistance to cumulative damage. Thanks to such mechanisms, the rocking pier survived, with no signs of structural distress, a deleterious sequence of seismic motions that caused collapse of the conventionally designed pier.

BACKGROUND AND OBJECTIVES

Capacity design, which forms the cornerstone of modern seismic design, aims at controlling seismic damage by strategically directing inelastic deformation to structural components, which are less important to the overall system stability [1]. Although this design approach is enforced or at least encouraged by modern seismic codes, it is conventionally limited to the superstructure. The foundation system is contrastingly treated conservatively. Specifically, the current foundation design leads to a strong unyielding foundation–soil system by adopting over strength factors to ensure that their ultimate capacity is reliably greater than the largest moment to be transmitted by the pier column.

Field evidence suggests that although new structures, complying with this capacity design rationale, are generally safer than the older ones, they remain vulnerable to very strong shaking. In fact, an alarmingly large number of modern structures have suffered intense damage leading to partial or total failure in recent earthquakes [e.g., [2, 3]] signaling the need to rethink the effectiveness of the aforementioned design practice. In response, a number of studies have explored the possibilities and constraints of an alternative design concept: allowing the development of 'plastic hinging' in the soil or at the soil–foundation interface, so as to reduce the possibility of damage to the elements of the structure.

Focusing on surface foundations, where nonlinearity manifests itself through uplifting and/or soil yielding, a 'reversal' of the current capacity design principle is proposed: the foundation is intentionally under designed under the seismic actions compared with the supported column to promote rocking response and accumulation of plastic deformation at the soil–foundation interface. Supporting evidence for this new approach has been provided by the following theoretical and empirical findings:

- Several theoretical and numerical studies on the rocking response of rigid blocks and elastic single-degree-of-freedom (SDOF) oscillators [e.g., [4-7]] provide compelling evidence that uplifting drastically reduces the inertial load transmitted into the oscillating structure.
- Because of the transient and kinematic nature of seismic loading, rocking response does not lead to overturning even in the case of very slender structures [8-14], except in rather extreme cases of little practical concern.
- Referred to as rocking isolation, allowing for foundation uplift has been proposed, and in a few exceptional cases employed in practice, as a means of seismic isolation [15-17].
- Even in the case of relatively heavily loaded footings or footings on soft soils, when rocking is accompanied with yielding of the supporting soil (and possibly momentary mobilization of bearing capacity failure mechanisms), substantial energy is dissipated in the foundation providing increased safety margins against overturning owing to the inherently self-centering characteristics and the ductile nature of rocking on compliant soil [e.g., [18-23]].
- Most importantly, a number of studies have recently investigated the scheme of rocking isolation, with emphasis on its effects on structures, which consistently point to a beneficial role of nonlinear foundation behavior for the overall system performance. Previous studies include the numerical and experimental work of

[24-26] in the domain of framed building structures, as well as those of [27-30] in the domain of bridges.

- A variety of modern numerical tools have been developed enabling comprehensive modeling of nonlinear rocking response [31-37], alleviating to some degree the skepticism regarding the uncertainties traditionally associated with prediction of the performance of rocking foundations for use in design.

On the basis of the exploratory work of [27], this study seeks to provide experimental verification of their numerical findings suggesting that although a conventionally designed reinforced concrete (RC) pier on an adequately large shallow foundation would suffer structural failure of the RC column and collapse in an earthquake sufficiently exceeding its design limits, rocking motion of an alternative under designed foundation would allow the same pier to survive even extreme shaking scenarios. To this end, it was necessary to realistically model the various attributes of nonlinear response of both the structural element (RC column) and the soil-foundation interface, therefore necessitating the use of the following:

- A new scale model reinforced concrete [38] capable of replicating stiffness, strength, failure mode, and post-failure response of the bridge pier.
- The University of Dundee (UOD) centrifuge earthquake simulator (EQS) to accurately replicate nonlinear soil behavior and provide repeatable replication of historically recorded earthquake motions.

A series of centrifuge tests were performed to investigate and compare the performance of the two RC model bridge piers, having the same structural section in each case, but each representing one of the two considered design approaches, namely, a conventional design and a rocking isolation design. Presented in this paper are the results from four of these tests involving a variety of shaking scenarios using real historical ground motions? Previous studies have simulated similar problems in the centrifuge by introducing reduced structural model cross sections (mechanical fuses) to control the locations of inelastic deformation and strength within structures usually made of aluminum [e.g., [29, 39]]. Despite the valuable insights of such testing, there are a number of unavoidable limitations:

- The location of any plastic damage must be defined a priori and any moment redistribution within the section is therefore suppressed.
- The axial load–moment capacity interaction behavior for aluminum is not of the same shape as for an RC section (in which axial load initially increases the moment capacity); therefore, any axial load redistribution within the section will not be captured correctly.
- The response of a fuse (whether in the form of notches or of artificial hinges) will not show the degradation in the moment–curvature behavior typical of reinforced concrete sections under cyclic loading.

The tests presented herein therefore attempt to model the inelastic response of RC elements in the centrifuge more realistically, namely, by using a new scaled model reinforced concrete consisting of a geometrically scaled steel reinforcing cage embedded within a cementations material, while simulating the entire soil–foundation–structure system as a whole. In this way, the correct foundation behavior can be modeled simultaneously with a structure having fidelity of response closer to that of a larger scale structural element test than has been achievable to date. Hence, the paper pursues two additional objectives: (i) presentation of the design and construction of realistically scaled RC model piers for use in centrifuge experiments and (ii) a report of 1–g calibration testing to verify the bending behavior of these piers. In the following, properties and results are given at prototype scale, unless otherwise stated.

DESIGN OF THE PIER–FOUNDATION SYSTEMS

Figure 1 illustrates the conceptual prototype of a modern bridge pier designed in accordance with Eurocode specifications for RC structures and seismic actions [40, 41]. The deck is a cast-in-place concrete box girder with a total dead load $q_1 = 200$ kN/m, free to rotate oscillating in the transverse direction. Thus, a 10.75 m tall (to the center of mass of the deck, including the footing) cantilever is considered carrying the concentrated mass of the supported deck ($m^{deck} = 300$ Mg) and comprising a RC column with a cross section of 1.5 m × 1.5 m (cross-sectional area $A_c = 2.25$ m2).

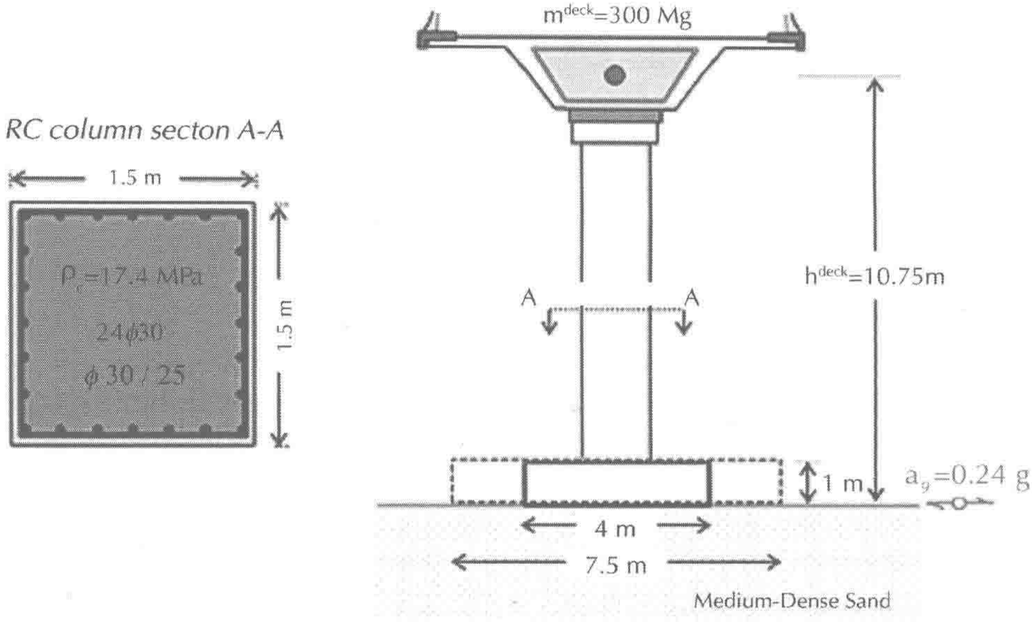

Figure 1: Schematic definition of the studied problem.

The RC column was designed to withstand static loads and seismic actions due to a design ground acceleration $\alpha_g = 0.24$ g. Using the EC8 specified acceleration response spectrum for a Type C soil profile and assuming ductile behavior (q = 3), the design spectral acceleration may be calculated as $S_A = 0.23$ g. Considering the availability of typical model reinforcement sizes and properties (discussed in the following section) as well as reasonable geometries to allow ease of construction, it was deduced that uniformly spaced longitudinal reinforcement of 24 bars of $d_{bl} = 30$-mm diameter combined with shear links of the same diameter spaced at 250 mm was required to effectively carry design loads, using a concrete of compressive strength $f_c = 17.4$ MPa (cylinder strength). Table 1 summarizes calculations regarding design loads (with reference to the base of the column) and verification of adequate reinforcement in shearing and bending (moment capacity and ductility). It should be noted that bending response was predicted through cross-sectional analysis employing the USC_RC software [42].

Table 1: RC column load calculations and design assessment with respect to the Eurocode

Loading and Resistance of RC section		
RC, reinforced concrete.		
Axial load	N:MN	3.426
Shear load	V:MN	0.677
Moment load	M:MNm	6.600
Normalized axial load	n (=N/$A_c f_c$)	0.088
Normalized shear load	ν (=V/$A_c f_c$)	0.017
Normalized moment load	m (=M/b$A_c f_c$)	0.112
Moment capacity	M_u:MNm	6.816
Factor of safety	FS	1.1
Curvature ductility	μ_φ	18
Displacement ductility	μ_Δ	6.5
Capacity design shear	V_{Ed}:MN	1.022
Shear reinf. yield shear force	$V_{Rd,s}$:MN	3.415
Maximum member shear force	$V_{Rd,max}$:MN	10.257
Shear resistance	V_{Rd}:MN	3.415
Factor of Safety	FS	3.3

The pier is founded on a shallow, 10-m thick, layer of medium density sand with a square (B × B) footing. Two different footing dimensions are studied, representing the two design alternatives. Table2 summarizes the two foundation designs listing: static and seismic loads (QE, ME), design seismic actions (QE_d, ME_d), and ultimate shear and moment capacities (Q_u, M_u) and the factors of safety for static vertical loads (FSV) and seismic loads (FSE). It should be noted that, apart from the small difference in the total vertical load N_{tot} (resulting from the different foundation sizes), the two piers are subjected to the same design loads (as they are identical in geometry) and to the same design ground acceleration α_g. QE_d and ME_d are calculated according to the capacity requirements for the foundation overstrength, which calls for seismic design actions on the foundation be substantially magnified (by as much as 40% in the case of a cantilever structure) in comparison with the actual seismic loads at the column base. Yet, only the conventional foundation complies with this requirement (hence having FS_E>1, as opposed to the rocking foundation which has FS_E<1). Combined with the limitations on the maximum allowable eccentricity ratio ($e = M/N < 2B/3$), this overstrength requirement leads to the conventional foundation being significantly larger ($B = 7.5$ m) than the rocking-isolated one ($B = 4$ m). Foundation capacity was, in a first step, calculated using the well-established failure envelope relationship of [43]

$$\left(\frac{Q}{t_h}\right)^2 + \left(\frac{M}{Bt_m}\right)^2 + 2C\frac{MQ}{Bt_h t_m} = \left\{\frac{N}{N_u}(N - N_u\right\}^2 \quad (1)$$

Where t_h, t_m, and C are parameters taken equal to 0.52, 0.35, and 0.22, respectively (determined through curve fitting of experimental results), and N_u is the ultimate capacity in pure vertical loading. In addition to theoretical predictions, the foundation capacities were also calculated using numerical simulations with finite elements (FE)—details are reported in [44].

Table 2: Foundation design: summary of actions and factors of safety (FS)

Breadth	B:m	7.5	4
Total vertical load	N_{tot}:MN	4.9	4
Seismic shear load	Q_E:MN	0.7	0.7
Seismic moment load	M_E:MNm	7.3	7.3
Design shear action	Q_{Ed}:MN	0.7	0.7
Design moment action	M_{Ed}:MNm	7.3	7.3
Ultimate shear capacity	Q_u:MN	1.2	0.5
Ultimate moment capacity	M_u:MNm	12.9	4.8
Factor of safety in vertical loading	FS_V	18	3.5
Factor of safety in combined (seismic) loading	FS_E	1.77	0.66

EXPERIMENTAL METHODS

The experimental program was carried out at the UOD and involved three different parts. First, eight RC model columns were tested in the standard four-point bending to optimize their construction procedure and verify their bending response. Second, two dynamic centrifuge model tests were carried out using elastic (aluminum) piers to measure the ultimate shear–moment capacity (Q_u–M_u) in the combined N–Q–M loading space for the two alternative foundations and check their M–θ response in comparison with FE predictions. In lieu of static pushover tests, this was achieved by exciting the soil–structure model with Ricker wavelets of suitable characteristics (PGA=0.6 g and dominant frequency f_E=1 Hz) causing the foundations to respond well beyond their nonlinear regime. Close approximation of the monotonic M–θ backbone curve was achieved, and the results were in good agreement with FE predictions. These characterization tests are reported in [44]. The third element of testing, which is the main scope of this paper, involved four centrifuge model tests, two for each of the alternative designs, wherein the RC model piers were subjected to different earthquake scenarios using real records of varying intensities. All tests were conducted at a scale of 1:50 (n=50) and at 50 g. The following section describes the modeling and testing procedures.

Reinforced Concrete Column Scaled Models: Construction and Validation

Modeling cementitious material at reduced scale is prone to size effects, owing to the presence of aggregates, and can lead to significant overstrength when the scale reduces substantially (n>10), as is usually the case in geotechnical centrifuge model testing. Therefore, this study has employed a novel scale model reinforced concrete, developed and validated by Knappett and co-workers [38, 45, 46], which realistically scales down both material stiffness and strength and reasonably replicates the response of RC structural elements under bending and shear loads. A gypsum-based mortar (beta-form surgical plaster) was used as a cementitious binder. Uniformly graded Congleton HST 95 silica sand [47] served well in modeling the aggregate phase of the concrete because its particle size distribution reasonably approximates the geometrically scaled grading curve of typical coarse aggregate at 1:50 scale. The plaster (P) and sand (S) were mixed with water (W) at a ratio of P/S/W=1:1:1 by weight. The compressive strength of the produced concrete model was measured through tests on 100×100 mm standard cubes as f_c,100=26.3 MPa (equivalent to cylinder strength of f'_c=17.4 MPa).

Reinforcement was modeled by stainless steel wire with a measured yield stress of $f_y = 460\,MPa$ (at 0.2% permanent strain), whereas the post-yield stress–strain response indicated significant hardening, making it closely representative of high yield reinforcement. The wire was cut and bent appropriately to produce scaled longitudinal bars (Figure 2(a)) and confining transverse reinforcement (Figure 2(b)). For ease of construction, wire of the same size, 0.6 mm in diameter at model scale (30-mm diameter at prototype scale), was utilized for both longitudinal and shear reinforcement. It should be noted that using shear links of smaller diameter, as is usually the case in practice, would require spacing lengths smaller than 5 mm at model scale to achieve the desired ductility, which would significantly hinder model fabrication. For both types of reinforcement, the wire was coated with HST95 silica sand using a fast-drying epoxy resin to produce a realistically rough interface between the steel and the model concrete.

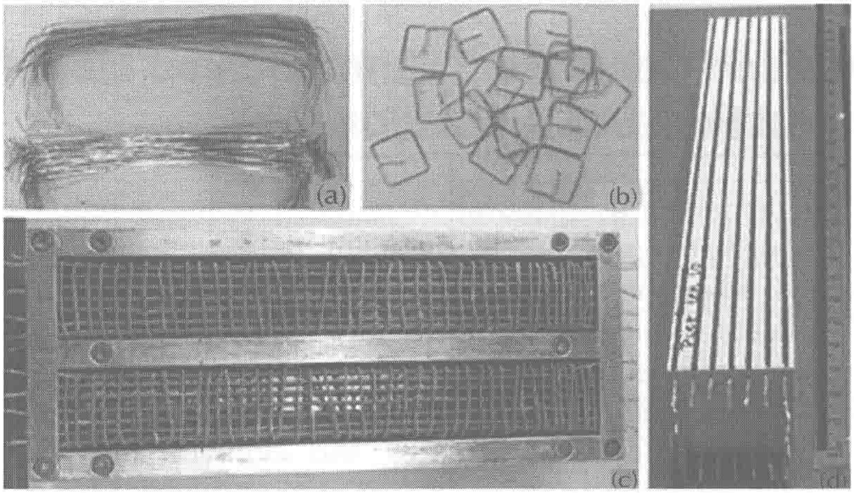

Figure 2: Construction of reinforced concrete model columns: (a) longitudinal steel with (upper) and without (lower) sand coating; (b) shear links; (c) steel within formwork; and (d) column model.

Fabrication of the reinforcement assembly was challenging because of the scale of the produced columns. The 200-mm long column model contained a total length of more than 5 m of wire modeling longitudinal reinforcement and 45 shear links uniformly spaced at a distance of approximately 5 mm. Anchoring of longitudinal reinforcement was achieved by providing an additional length of about 10 mm on each side of the column, which was bent and fixed within the foundation or the deck plates. Test units were cast in a custom-built formwork (Figure 2(c)), which allowed casting of two columns at a time. The column models (Figure 2(d)) were left to cure for at least 2 weeks before testing.

A total of 19 columns were produced, most of which were used in bending tests intended to verify the moment–curvature response of the column section and the capacity of the column–foundation joint. It is important to note that in addition to shear links, the alternative of using continuous spiral shear reinforcement was also investigated but was found less effective in producing sections with a constant core area resulting in less accurate replication of the cross-sectional capacity.

A series of standard four-point bending tests were carried out under displacement control. Figure 3(a) shows a typical test on a column model identical to those used in the centrifuge, highlighting the loading arrangement and the typical mode of failure. It can be seen that the observed crack pattern is typical of a column designed to fail in flexure (i.e., containing sufficient shear reinforcement to suppress shear failure). This was expected considering the calculated moment and shear capacities of the section given in Table 1. Vertical cracks may be identified within the 60-mm wide central span, which is subjected to pure bending (constant moment) while their inclination evidently changes in the two zones of combined moment□shear loading between the load points and supports. Figure 3(b) compares the measured bending behavior of the column section (in prototype scale), in terms of bending moment (M) versus deflection ( ), with numerical predictions using USC-RC for pure bending conditions (without axial load) indicating very satisfactory agreement. Unfortunately, the available laboratory equipment did not allow testing with axial load to measure the effect of the pier weight on the section response. Yet, the M□δ response of the concrete section under axial load equal to the pier weight (N = 3.4 MN) was calculated numerically and is also shown in Figure 3(b). As expected, the weight of the pier, which is present in the centrifuge model tests, results in considerable increase of the section moment capacity, although at the cost of reduced ductility. It is important to note that the USC-RC-predicted moment capacity for the actual axial load is in good agreement with the maximum moment load recorded in the centrifuge tests.